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THE ROLE OF THE WAVE EQUATION IN RATIONAL DESIGN OF PILE FOUNDATIONS

by

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Soil Mechanics Bureau

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NEW YORK STATE DEPARTMENT OF TRANSPORTATION
SOIL MECHANICS BUREAU

"THE ROLE OF THE WAVE EQUATION
IN RATIONAL DESIGN OF PILE FOUNDATION"

RATIONAL PILE FOUNDATION DESIGN PROCEDURE

I. SUBSURFACE CONDITIONS DETERMINATION

1. Perform subsurface explorations and construct a soil profile including layer boundaries and possible obstructions, man-made or natural.

2. Assign engineering parameters to each soil layer using all available subsurface information, after completion of a testing program. (Friction angle, cohesion, density, etc.)

3. Identify and classify zones in the soil profile according to whether they are favorable or unfavorable with respect to foundation performance, i.e. a) Favorable: Strong, relatively incompressible, b) Unfavorable: Weak, compressible.

II. LOAD ANALYSIS

Determine the nature and magnitude of the loads to be supported and the probability of their occurrence to facilitate the selection of a foundation design.

III. ALTERNATE DESIGNS

At this point, reconsider all other foundation types and treatments which would provide a viable alternate to a pile foundation.

IV. FRICTION PILE ANALYSIS

1. A qualitative analysis of potentially suitable pile types is made by successively evaluating support capacity in each of the favorable zones of the subsurface profile. Include an evaluation of detrimental effects of all overlying (Negative Skin Friction) and underlying (settlement) unfavorable zones. Eliminate all obviously unsatisfactory

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alternatives. For each appropriate pile type choose a diameter and reasonable load per pile and calculate a pile length using a static analysis. Determine if this pile length and type satisfies the design requirements for deflection and moment under the applied lateral load. (See Appendix A)

2. Estimate the installation problems to be encountered for each particular pile type, i.e. vibration, damage to existing structures, noise pollution requirements, pile heave, jetting, corrosion, discontinuities, etc., and eliminate unsuitable alternatives.

3. Make an economic comparison of remaining suitable alternatives using a cost per pile or a cost per ton-ft. of piles as the economic factor. Choose final designs.

4. Determine if pile load tests (Static, Constant Rate of Penetration, or Dynamic) are needed. If so include the procedure in the contract documents and decide whether to drive the test pile to a resistance or an elevation.

5. Establish necessary restrictions on installation; Tolerance requirements, jetting limitations, limitations on the use of spuds or followers. Include a generalized soil profile including relevant test data (See Figure 1) and all pertinent special notes to alert the Contractor of unusual foundation conditions or special requirements. (See Appendix B)

V. END BEARING PILES

In the event rock or a suitable dense layer is shallow enough to reach economically, End Bearing Piles should be considered for use. The design pile load is then established based on the bearing capacity of the dense soil or rock, the structural capacity of the pile and lateral load requirements.

VI. DESIGN REVIEW

A foundation engineer (preferably the engineer involved) should review the contract proposal and plans to determine if the contract documents are clear as to what the Contractor must do.

VII. CONSTRUCTION CONTROL

After a project has been let for construction the Contractor must submit details of the pile driving equipment he intends on using (See Figure 2). We then either accept or reject this equipment based on wave equation analyses which test the capability of the equipment to drive the designated pile to the estimated length without damage. If the driving system is adequate, we then will specify the required blow count and other pertinent data to the Contractor.

VIII. APPLICATION OF STATIC ANALYSIS AND WAVE EQUATION ANALYSIS FOR CONSTRUCTION CONTROL

Wave equation analyses for construction control are generally begun on receipt of a Pile and Driving Equipment Data form (See Figure 2). This form is completed by the Contractor and submitted to the State at least 2 weeks prior to starting pile driving. This information is combined with previously determined soil properties and static analyses to perform wave equation analyses.

A. Typical Wave Equation Analysis for a Friction Pile

Our most common usage of the wave equation analysis is to determine driving criteria for friction piles. The output from this analysis is compared with actual field results to determine if the design capacity is being attained at the estimated length. If the actual blow count is less than the predicted, an immediate review must be made of all facets of the design to determine cause. Commonly encountered problems involve underestimating temporary remolding of the subsoil or only analyzing the initial pile driven when the design has included the effect of densification due to pile group installation.

Figures 3A, B, and C-1 through C-8 illustrate a simple typical friction pile analysis and wave equation input and output. The bearing graph shown in Figure 3C-8 is of most interest to field personnel because of the obvious relationships between capacity, stroke, blow count, and stress.

B. Typical Wave Equation Analysis for an End Bearing Pile

For many years little attention was paid to installation of end bearing piles other than obtaining the established blow count criteria for "refusal" or "practical refusal". This lack of knowledge regarding the destructive capability of the hammer-pile system led to many damaged piles which were unnecessarily overdriven.

Figures 4A, B, and C-1 through C-8 illustrate a simple typical end bearing pile analysis. Note in Figure 4C-8 the rapid increase in stress with increasing stroke and blow count.

At the time of driving certain subsurface conditions may exist that will greatly influence the required blow count obtained from the wave equation. Additional static analyses will be required to provide proper input to the wave equation. Commonly encountered situations and methods of solution are described below.

A. Soils Subject to Remolding During Pile Driving

During pile driving certain types of soil, namely silts and clays, exhibit a temporary loss of strength. The following procedure should be used to account for this phenomenon and the resulting low pile blow counts. (Figure 5)

1. Perform static analysis as in Appendix A to determine the length required for design capacity.

2. Using this length and remolded soil strengths (see Refs. 11 and 12) recompute the soil resistance along the pile and the capacity.

3. Input the remolded resistance and capacity values into the wave equation to determine required pile blow count at time of driving.

4. If the amount of soil strength regain (set-up) is in question, a retap of a previously driven pile should be made and the resulting blow count compared with wave equation results from the original static analysis and actual design capacity. The duration of time between driving and retap should be based on an analysis of the drainage properties of the soil.

B. Soils Subjected to Scour

The foundations for structures at water crossings must be designed to sustain extreme flooding which will cause erosion or scour at piers. Therefore, piles are designed to achieve the design load below the depth of scour. However, the pile blow count at the time of driving will reflect penetration of the soil within the zone of scour. (Figure 6)

1. Estimate depth of maximum anticipated scour. (See Ref. 13)

2. Perform static analysis to determine pile length for required design capacity assuming zero soil resistance to scour depth.

3. Using that pile length, recompute actual soil resistance and actual pile capacity by including soil resistance within scour zone.

4. Input actual soil resistance and actual pile capacity into wave equation to determine required pile blow count at the time of driving.

C. Unfavorable Soil Deposit

Pile foundations often must completely penetrate deposits of favorable soil (or recent fill) and unfavorable soil which overlie the bearing stratum. Although these overlying layers do not contribute soil resistance to the design pile capacity, this resistance must be considered at the time of pile driving unless preaugering is utilized. (Figure 7)

1. Perform static analysis to determine length of pile required in the proposed bearing stratum. Then determine total length.

2. Using that length recompute soil resistance including overlying soil layers and determine resulting capacity at time of pile driving.

3. Input the recomputed values into the wave equation to determine required blow count.

APPENDIX A

I. STATIC ANALYSIS FOR PILE BEARING CAPACITY

A. Bearing Capacity of a Single Pile - For each pile type under consideration a length will be determined, based upon the ultimate strength of the pile-soil system.

1. Structural capacity of the Pile - Determine the allowable load on the pile based on structural considerations alone, e.g. building code stresses. See Table 13-1 in Ref. #4 or Ref. #1.

2. Bearing Capacity of the Soil - A pile length is determined based on the ultimate shear strength of the soil using a safety factor of 2. The method of calculating the bearing capacity is based on soil type.

   a. Cohesionless Soil (c=o) - In granular materials two methods of analysis are used; for preliminary lengths use the approach as found in Broms (Ref. #2), for final lengths use Nordlund's Analysis (Ref. #8).

      1. Preliminary (Ref. #2)

         a. End Bearing in TSF=2.5N(Stand. Penetration Resistance)
         b. Skin Friction in TSF=.02 N

      2. Nordlund's Analysis - This method is the best available for cohesionless soils. It accounts for more factors than any other equation. Densification of the soil due to pile driving, overburden reduction, pile volume, pile material, pile taper all these factors can be included in the calculation of bearing capacity. (See Ref. #8)

   b. Cohesive Soils (Ø=o) - In cohesive soils a preliminary method is used based on the Standard Penetration Test and a more detailed method based on an empirical method by Tomlinson.

1. Preliminary - Undrained Shear Strength = N/8 (Ref. #2)

2. Tomlinson - The adhesion of clay soils to piles is based on pile material and the strength of the clay. The stronger and stiffer the clay the more the adhesion is reduced relative to the shear strength of the soil (Ref. #10, Ref. #1 and Ref. #7). Also, in a layered system see Ref. #9., Tomlinson has taken into account the intrusion of upper layers carried into lower ones. Shear strength is based on a conservative interpretation of Laboratory and Field Testing Programs.

3. As a check, Broms Ref. #2 can be used to provide a range of results. These adhesion values are very conservative.

c. Silty Soils (Ø & c soils) Generally silts are analyzed similar to clays, using Tomlinson's analysis. However, the results should be checked with a drained analysis, (See Ref. #12, Chap. 21) assuming a drained friction angle.

B. Bearing Capacity of Pile Groups - This analysis is subdivided by soil type but in general the capacity of a pile group depends on the pile spacing.

1. Cohesive Soils - As per Ref. #7 the efficiency factor varies between 0.7 at a pile spacing of 3 pile diameters, to 1 at a spacing equal to 8 pile diameters. For spacings less than 3 the group fails as a block.

2. Cohesionless Soils - For sands and gravels and efficiency factor of 1 is a good design value as in Ref. #7.

II. SETTLEMENT

Settlement of a Pile Foundation - Settlement prediction is perhaps the least accurate of the aspects of pile design. For long term settlement of a pile group, the 2/3 rule method as outlined in NAVDOCKS Ref. #4 is used.

III. NEGATIVE SKIN FRICTION

Negative skin friction is caused by a relative downward movement of the soil surrounding a pile. All available downdrag or negative skin friction will be mobilized whenever there is more than 0.4" relative movement between pile and soil in cohesionless soils and 0.2" relative movement in cohesive soils. Therefore, whereever such movements can occur, negative skin friction should be accounted for. Please note that although Tomlinson's adhesion values are conservative when used in bearing capacity analyses, unconservative negative skin friction values will result unless the adhesion values are increased. At present NAVDOCK'S Ref. #4 gives the most general method for determining negative skin friction. However, for end bearing piles Garlanger (Ref. #6 and Fellenius (Ref. #5) are promising methods.

IV. LATERAL LOAD ANALYSIS

The relationships between applied lateral load, pile moment and deflection, and soil parameters are determined for a single pile by a method developed by Reese et al (References 14, 15, 16, 17). This method includes many pertinent design variables not accounted for in other methods, although a computer solution is required to incorporate all variables in the analysis. The computer program (code name COM 622) developed by Reese is available from the Computing Center, University of Colorado, Boulder, Colorado 80302. More general cases with no axial loads and constant pile stiffness may be readily solved by hand using non-dimensional coefficients in the aforementioned publication or by Broms (References 18, 19). As of now, no reliable method has been developed to ascertain pile group effects. The most promising method for groups was developed by Poulos (Reference 20).

APPENDIX B

TYPICAL SPECIAL CONTRACT NOTES FOR PILE DRIVING (NYS DOT)

1. "Piles will be acceptable only when driven to pile driving criteria established by the Deputy Chief Engineer (Structures). Prerequisite to establishing these criteria, the Contractor shall submit, to the Deputy Chief Engineer (Structures) and others as required, Form BD 138, 'Pile and Driving Equipment Data'. All information listed on Form BD 138 shall be provided within fourteen (14) days after the award of the contract. Each separate combination of pile and pile driving equipment proposed by the Contractor will require the submission of a corresponding Form BD 138."

2. "It is possible that difficult driving of piles may be encountered and it may be necessary to utilize mechanical equipment for removing consolidated material or boulders from the location of piles. This may be accomplished by various types of earth augers, well drilling equipment, or other devices to remove the consolidated material to permit piles to be driven to the desired depth or rated resistance without distortion."

3. "If any obstructions to pile driving are encountered ten (10) feet or less from the bottom of the footing, the Contractor shall, if so ordered by the Engineer, pull the partially driven pile or piles and remove the obstruction, backfilling the hole with approved suitable material which shall be thoroughly compacted to the satisfaction of the Engineer. However, no partially driven pile shall be removed until the Engineer is satisfied that the Contractor has made every effort to drive the pile through the obstruction. Payment for the excavation will be made at the unit price bid for the Structure Excavation Item and for the temporary sheeting under Item _________ when sheeting is used. No other extra payment will be made for this work."

4. "The ordered length of pile shall be measured below the cut-off elevation shown on the plans. Any additional lengths of pile or splices above the cut-off elevation necessary to facilitate the Contractor's operation shall be at his own expense."

5. "Piles for __________ are driven because of possible scour of stream bed and shall be driven to the minimum lengths shown on the plans regardless of the resistance to driving."

6. "The Contractor's attention is directed to the tip elevations on rock for the H-piles which will laterally support ________. The hammer provided to drive these piles must be capable of achieving the required penetration through the compact over-burden which may present hard driving conditions. To attain the necessary lateral resistance, these piles must penetrate to the ledge rock surface. Therefore, prior to pile driving approval of the Deputy Chief Engineer (Structures) will be required in accordance with Section 629 of the specifications. This approach shall be based on a review and evaluation of data submitted by the Contractor on Form BD 138."

7. "A static pile load test shall be performed at a location designated on the plans, or as specified by the Deputy Chief Engineer (Structures) in accordance with SCP-5 the Static Pile Load Test Manual. The pile load test shall consist of either the constant rate of penetration test or the maintained load test as specified. It shall be performed after a minimum seven (7) day waiting period. The Contractor shall prevent the test pile from rebounding during the waiting period and prior to application of the test loads.

8. "Dynamic load tests will be conducted by representatives of New York State on the Static Load Test Pile and at least one pile in each pier, or at other locations ordered by the Engineer. The Contractor shall furnish the appurtenant construction equipment necessary to perform the field tests in accordance with 'Furnishing Equipment for Dynamic Load Testing of Piles' Item."

9. "Piles for the existing structure should be removed where they interfere with the pile driving for the new structure."

10. "It shall be the Contractor's responsibility to place the cofferdams for ________ so that they will not interfere with the driving of batter piles. Pay lines for the cofferdams shall be as shown on the plans."

11. "The general subsurface conditions at the site of this structure are as shown on Drawing No. ___________."

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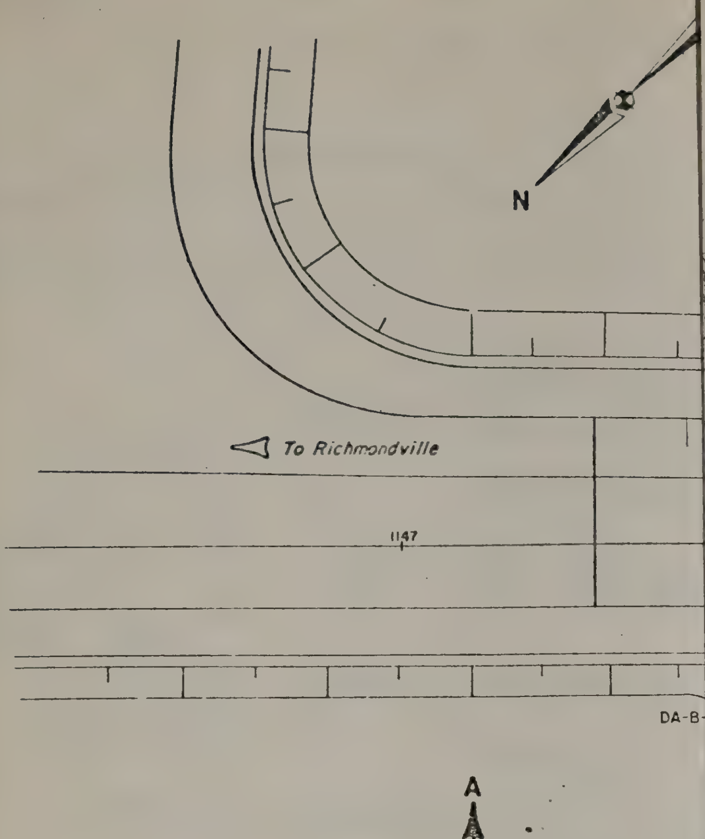
FED RD REG NO	STATE	FEDERAL AID PROJECT NO	SHEET NO
	NEW YORK		

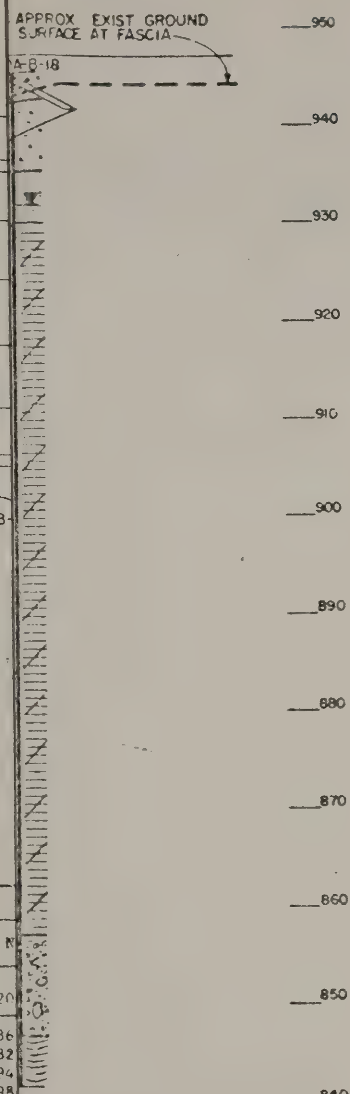
INTERSTATE ROUTE 508 (I-88)
RICHMONDVILLE-COBLESKILL

CAPITAL PROJECT IDENTIFICATION NUMBER 935714



PLAN

SCALE: 1"=20'



LABORATORY GRAIN SIZE DISTRIBUTION TEST SUMMARY

Drill Hole No.	Representative Sample Depth (Ft.) (See Note 3)	% Passing By Weight U.S. Standard Sieve N							
		1"	1/2"	1/4"	10	20	40	100	20
DA-B-18	0-1.5	100	97.6	94.9	86.3	76.9	66.0	45.0	36
"	5.0-6.5	100	90.0	69.5	52.8	45.1	40.8	35.4	32
"	10.0-11.5	100	100	100	99.9	99.7	99.3	97.6	94
"	15-16.5 & 20-21.5	100	100	100	100	99.9	99.8	99.1	98
DA-B-19	0-1.5 & 5-6.5	100	[illegible]	[illegible]	55.7	47.4	47.4	[illegible]	30
"	10-21.5 & 15-16.5	100	100	80.4	72.3	68.3	66.3	60.2	55
"	20-21.5	100	100	100	99.9	99.7	99.6	99.5	99

NOTES:
1. The soil samples used for these tests were extracted with a 1½ inside diameter sample spoon.
2. The test method used for these grain size analyses conforms to D422 as modified by Bureau of Soil Mechanics Technical Manual
3. Where samples from more than one depth are noted, it was neces samples of insufficient material for individual grain size tes

REFERENCE PLANS

Preliminary Structure Plans Used for Analysis were

Prepared By: The Structures Design and Construction Subdivision.

Scale	Date
1"=20'	Sept 5, 1975

Prepared By R P Ungerer
Drawn By: Joe Mailor
Dwg Reviewed By J. J. M...
Checked By Robert C Houghton

GENERA

The subsurface explorations shown hereon wer
Regional Soils Section
1) General soil and rock (where encountered) are based on an engineering interpretation o Soil Mechanics Bureau and may not necessaril conditions between borings and samples. Det ditions encountered in individual borings ar
2) The observed water levels and/or conditio recorded at the time of exploration. These siderably, with time, according to the preva are otherwise dependent on the duration of a
3) Sound engineering judgment was exercised sented hereon. This information was prepared purposes only. Its presentation on the plans intended users with access to the same infor information interpretation is presented in g for personal investigation, independent inte
4) All structure details shown hereon are fo indicative of the final design conditions sh
5) Footing elevations shown are as indicated

Clay.
Silty
ble

FIGURE 1

APPROVED Oct. 17 1975
L. H. Moore
DIRECTOR
SOIL MECHANICS BUREAU

REGION NO. 9
COUNTY Schoharie
DWG NO 9 SM 1848

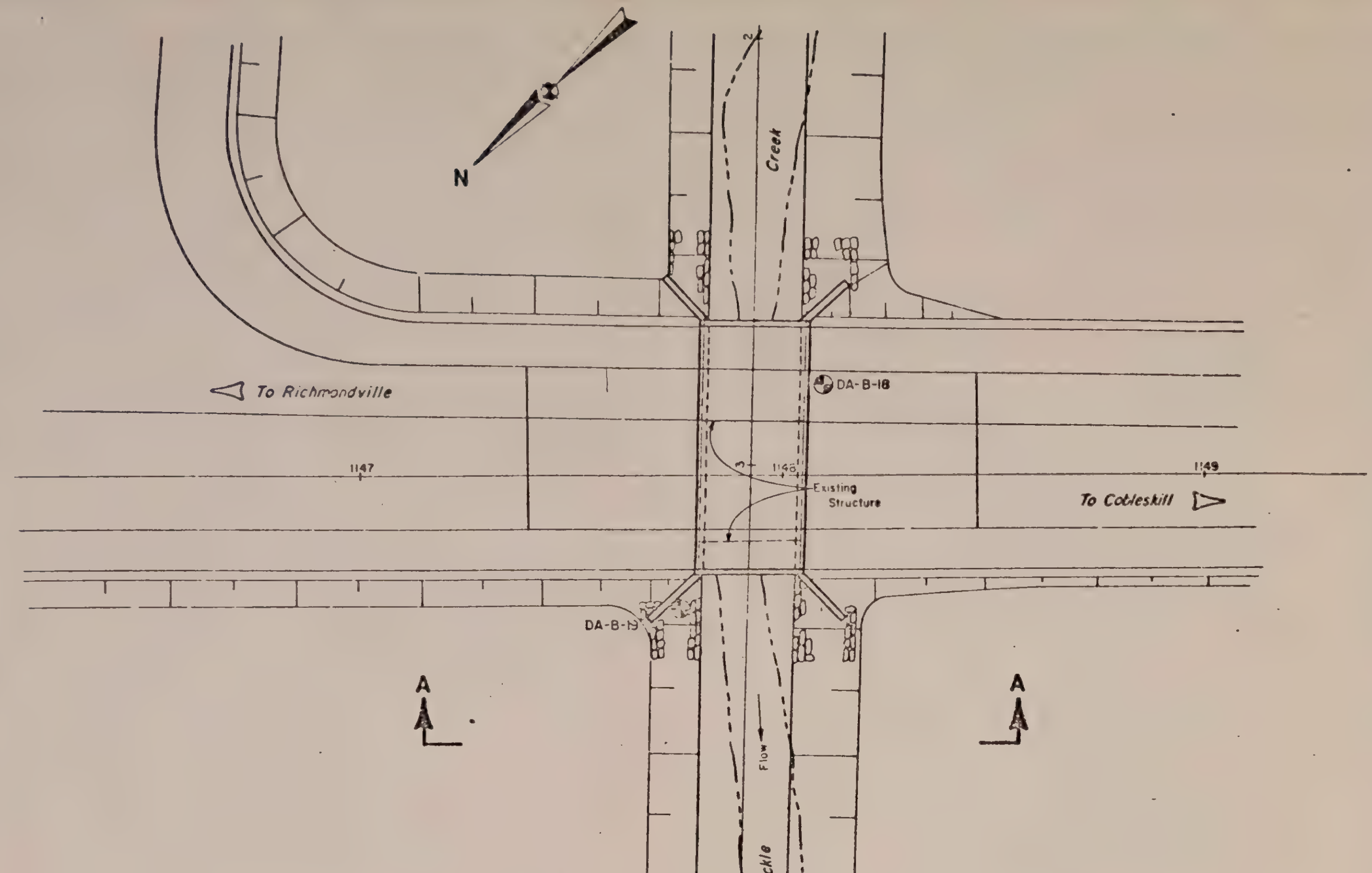
STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION
DESIGN AND CONSTRUCTION DIVISION

GENERAL SUBSURFACE PROFILE F
BRIDGE NO.6 ROUTE 7 OVER
MICKLE CREEK

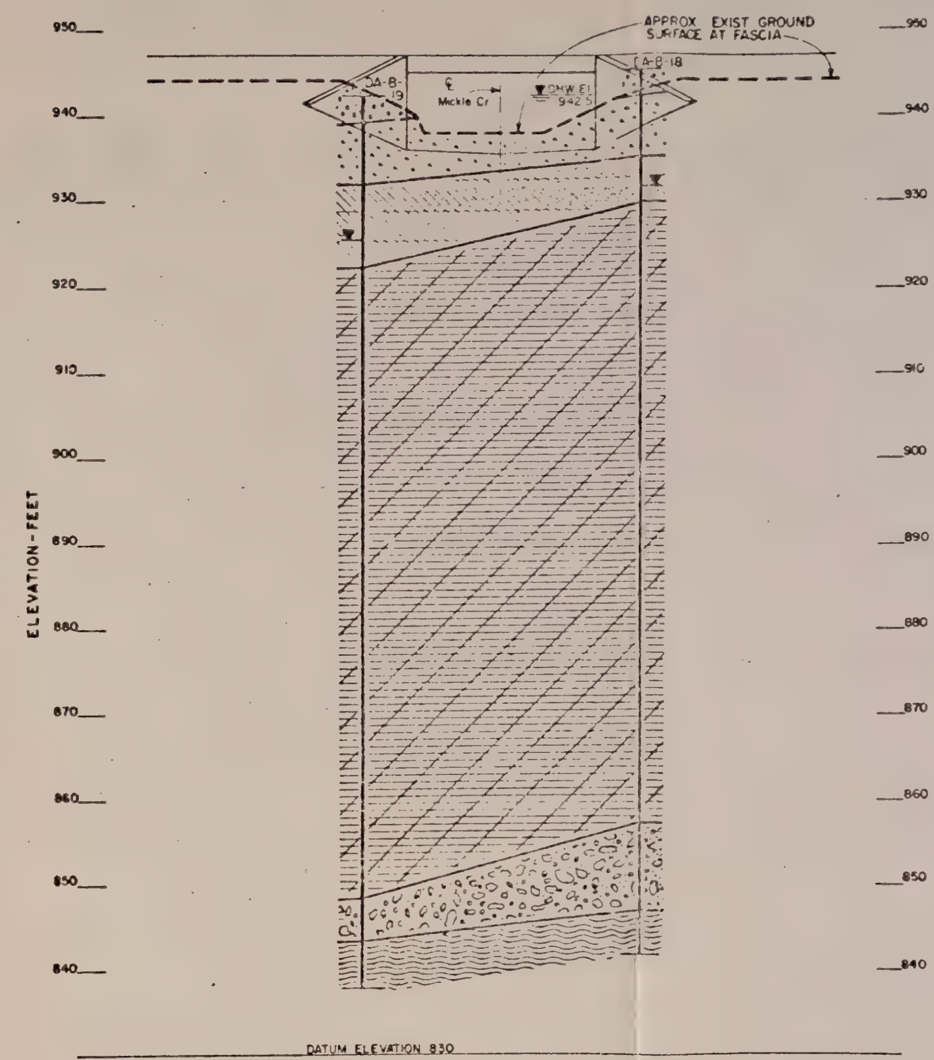
FED RD REG NO	STATE	FEDERAL AID PROJECT NO	SHEET NO
	NEW YORK		

INTERSTATE ROUTE 508 (I-88)
RICHMONDVILLE-COBLESKILL

CAPITAL PROJECT IDENTIFICATION NUMBER 9357.4



PLAN
SCALE 1"=20'



ELEVATION A-A
BORINGS PROJECTED PERPENDICULAR TO SECTION LINE
SCALE 1"=10'

LABORATORY GRAIN SIZE DISTRIBUTION TEST SUMMARY

Drill Hole No.	Representative Sample Depth (Ft.) (See Note 3)	% Passing By Weight U.S. Standard Sieve No.								Hydrometer Analysis .002 mm
		1"	1/2"	1/4"	10	20	40	100	200	
DA-B-18	0-1.5	100	97.6	94.9	86.3	76.9	66.0	45.1	36.4	6.8
"	5.0-6.5	100	90.0	69.5	52.8	45.1	40.8	35.4	32.4	6.7
"	10.0-11.5	100	100	100	99.9	99.7	99.3	97.6	94.9	21.5
"	15-16.5 & 20-21.5	100	100	100	100	99.9	99.8	99.1	98.7	59.9
DA-B-19	0-1.5 & 5-6.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	30.9	6.8
"	10-11.5 & 15-16.5	100	100	80.4	72.3	68.3	66.3	60.2	55.6	18.3
"	20-21.5	100	100	100	99.9	99.7	99.6	99.5	99.4	52.4

NOTES:
1. The soil samples used for these tests were extracted with a 1½" nominal inside diameter sample spoon.
2. The test method used for these grain size analyses conforms to ASTM Designation D422 as modified by Bureau of Soil Mechanics Technical Manual No. TM(s) 64-2.
3. Where samples from more than one depth are noted, it was necessary to combine samples of insufficient material for individual grain size tests.

REFERENCE PLANS

Preliminary Structure Plans Used for Analysis were Prepared By The Structures Design and Construction Subdivision.

Scale	Date
1"=20'	Sept 5, 1975

Prepared By P. Unger
Drawn By Joe Naylor
Dwg Reviewed By J. M.
Checked By Robert C. Hayth

GENERAL NOTES

The subsurface explorations shown hereon were made between May 29,1975 to June 9,1975 by the Regional Soils Section

1) General soil and rock (where encountered) strata descriptions and indicated boundaries are based on an engineering interpretation of all available subsurface information by the Soil Mechanics Bureau and may not necessarily reflect the actual variation in subsurface conditions between borings and samples. Detailed data and field interpretations of conditions encountered in individual borings are shown on the subsurface exploration logs.
2) The observed water levels and/or conditions indicated on the subsurface profiles are as recorded at the time of exploration. These water levels and/or conditions may vary considerably, with time, according to the prevailing climate, rainfall or other factors and are otherwise dependent on the duration of and methods used in the explorations program.
3) Sound engineering judgment was exercised in preparing the subsurface information presented hereon. This information was prepared and is intended for State design and estimate purposes only. Its presentation on the plans or elsewhere is for the purpose of providing intended users with access to the same information available to the State. This subsurface information interpretation is presented in good faith and is not intended as a substitute for personal investigation, independent interpretations or judgment of the Contractor.
4) All structure details shown hereon are for illustrative purposes only and may not be indicative of the final design conditions shown in the contract plans.
5) Footing elevations shown are as indicated at the time of this drawing's preparation.

LEGEND

The following tables summarize the descriptive information used on this profile

Density (Non Plastic Soils)	No. of blows per foot of penetration of 2 inch O.D. (1-1/2 inch I.D.) sampler using a 300 lb drop hammer, 18 inch fall
Very Loose	0-3
Loose	4-8
Medium Compact	9-20
Compact	21-35
Very Compact	over 35
Consistency (Plastic Soils)	
Very Soft	0-2
Soft	3-6
Firm	7-12
Stiff	13-20
Hard	over 20

The system for describing soil materials shown on this drawing is detailed in "An Engineering Description of Soils, Visual-Manual Procedure" Official Issuance No. 741-5 STP 2/74 prepared by the New York State Department of Transportation

SYMBOLS

- DA-B-__; Drill Holes
- Observed Water Level
- Loose Brown Gravelly SILT, with Fibers.
- Loose to Compact Brown Gravelly SILT, with Clay.
- Firm Brown Clayey SILT, with Sand.
- Very Soft to Soft Gray Silty CLAY.
- Compact to Very Compact Gray and Brown Silty GRAVEL, with Clay.
- Very Compact Black Silty Decomposed Shale

FIGURE 1

APPROVED Oct. 17 1975
L. H. Moore
DIRECTOR
SOIL MECHANICS BUREAU

REGION NO. 9
COUNTY Schoharie
DWG NO. 9 SM 1848

STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION
DESIGN AND CONSTRUCTION DIVISION

GENERAL SUBSURFACE PROFILE F
BRIDGE NO.6 ROUTE 7 OVER
MICKLE CREEK

DRAWING NO. 1 OF 5

PILE AND DRIVING EQUIPMENT DATA

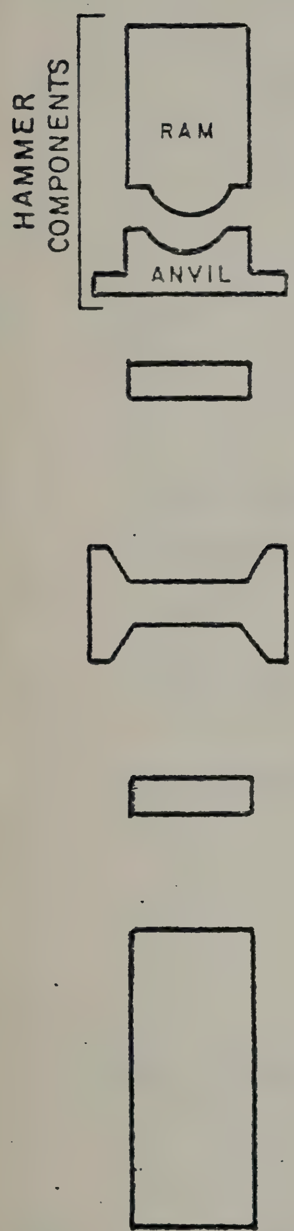
P. I. N.: __________ Contract No.: __________ Structure Name and/or No.: __________

Project: __________

__________ Pile Driving Contractor or Subcontractor: __________

County: __________ __________

(Piles driven by)



HAMMER

Manufacturer: __________ Model __________

Type: __________ Serial No.: __________

Rated Energy: __________ @ __________ Length of Stroke

Explosive Force: __________
(For diesel hammers)

RAM

Ram Weight: __________ Ram Length: __________
(For diesel hammers)

Ram Cross Sectional Area: __________

ANVIL

(With diesel hammers) Anvil Weight: __________

CAPBLOCK

Material: __________ Area: __________

Thickness __________

Modulus of Elasticity - E __________ (P.S.I.)

Coefficient of Restitution-e __________

PILE CAP – Helmet / Bonnet / Anvil Block / Drivehead – Weight: __________

CUSHION

Cushion Material: __________ Area: __________

Thickness: __________

Modulus of Elasticity - E __________ (P.S.I.)

Coefficient of Restitution-e __________

PILE

Type: __________

Pile Size: Length (In Leads)- __________

Diameter- __________

Wall Thickness: __________ Taper: __________

Material: __________ Weight/Ft.: __________

Design Pile Capacity: __________ (Tons)

Description of Splice: __________

____

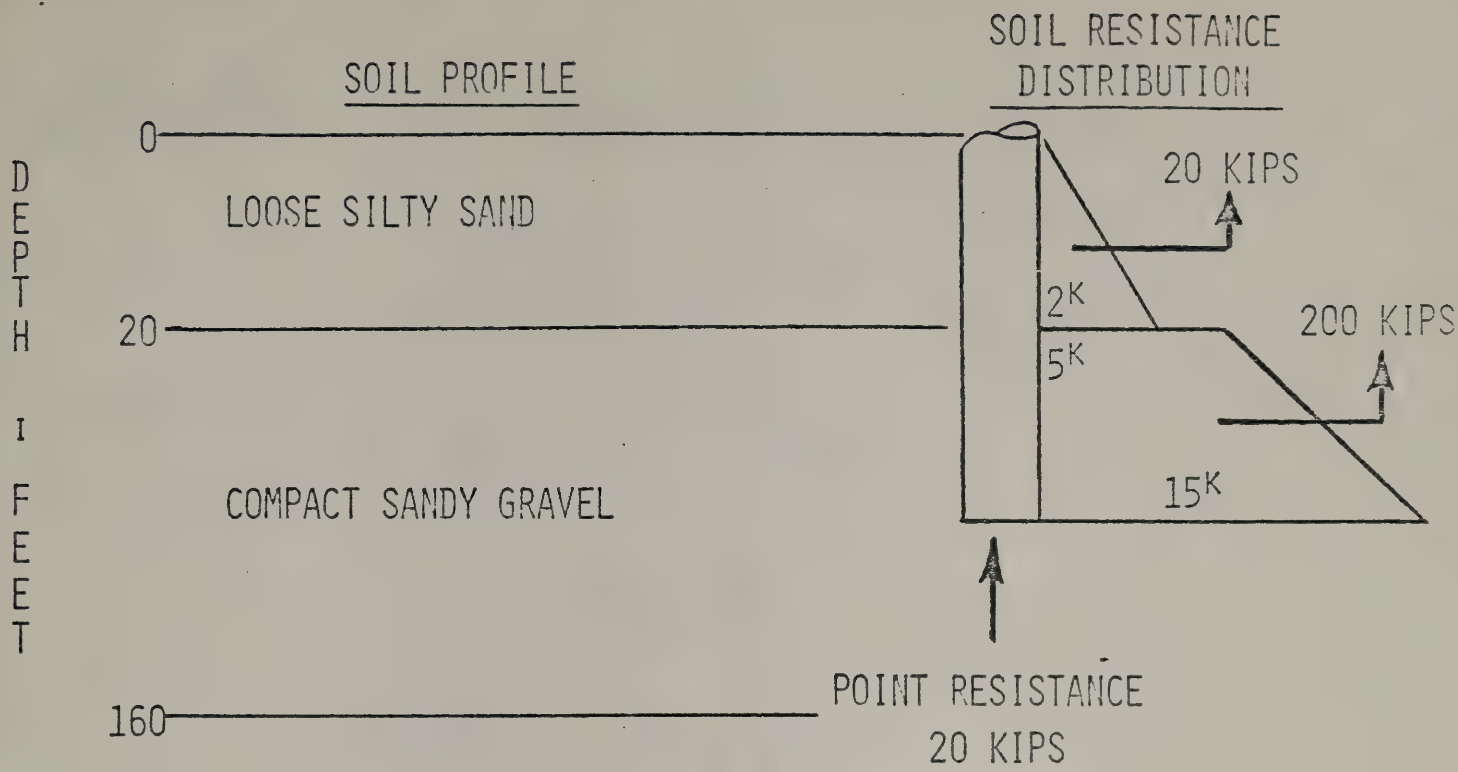
Tip Treatment Description: __________

____

NOTE: If mandrel is used to drive the pile, attach separate manufacturer's detail sheet(s) including weight and dimensions.

Submitted By: __________ Date: __________

FIGURE 2



PILE DESIGN LOAD	60 TONS
ULTIMATE SOIL RESISTANCE (SAFETY FACTOR = 2)	120 TONS
LENGTH FOR 120 TON RESISTANCE	40 FEET
PILE TYPE - - - 12-3/4 INCH DIAMETER PIPE,	0.250 INCH WALL

CASE DAMPING VALUES

	SKIN	LAYER 1 = 0.4 LAYER 2 = 0.2	USE 0.2
	TOE		0.1
QUAKE VALUES	SKIN		0.1
	TOE		0.1

DESIGN DATA FOR WAVE EQUATION ANALYSIS

FRICTION PILE EXAMPLE

FIGURE 3A

INPUT CARD REFERENCE NO.	DESIGNATION OF NECESSARY INPUT				
	TITLE				
1.0	FRICTION PILE				
	IOUT	IHAMR	IPERCS		
2.0	10, ,	4, ,,,,,,,,	92 *		
	WT. CAP	STIFF-CB			
3.0	1.2,	21,000., *			
		C.O.R.-CB			
4.0	,	0.8, *			
	LENGTH	AREA-PILE TOP			
5.0	40.0,	9.8, *			
	QUAKE-SKIN	QUAKE-TOE	DAMP-SKIN	DAMP-TOE	RULT
6.0	0.1,	0.1,	0.2,	0.1,	-1.0,*
	X_{PI}	% SOIL RESISTANCE			
6.401	0.0	0.0			
6.402	20.0	2.0			
6.403	20.0	5.0			
6.404	40.0	15.0			
7.0	BLANK				
8.0	BLANK				
	ULTIMATE RESISTANCE				
9.0	60.0,	90.0,	120.0	180.0,	240.0, *

CASE WAVE EQUATION INPUT

FRICTION PILE EXAMPLE

FIGURE 3B

FRICTION PILE

PILE DESCRIPTION

X REL. TOP (FT)	0.0	40.0
A (SQ. IN.)	9.8	9.8
E (KSI)	29000.	29000.
GAMMA (LB/CU FT)	492.0	492.0

HAMMER MODEL DEL. D-22

ELEMENT NUMBER	WEIGHT (KIPS)	STIFFNESS (K/IN)	COFF. RESTITUTION
1	1.213		
2	1.213	191666.7	
3	1.213	191666.7	
4	1.213	191666.7	
ANVIL	1.595	101833.3	0.850
CAP	1.200	21000.0	0.800
CUSHION		0.0	1.000
PILE TOP			0.850

PILE PROPERTIES

PILE LENGTH= 40. FT., AREA(AT TOP)= 9.8 S-IN
E. MODUL(AT TOP)=29000. KSI., SPEC. WT.(AT TOP)= 492. LBS/CU FT

NO.	WEIGHT (KIPS)	STIFFN. (K/IN)	PDAMP. (KS/FT)	SPLICE (KIPS)	SOIL-S (PCT.)	SOIL-D (KS/FT)	QUAKE (IN.)	L.B.T. (FT.)
1	0.149	5340.	0.34	0.	0.004	0.015	0.100	4.4
2	0.149	5340.	0.34	-5000.	0.012	0.046	0.100	8.9
3	0.149	5340.	0.34	-5000.	0.021	0.077	0.100	13.3
4	0.149	5340.	0.34	-5000.	0.029	0.108	0.100	17.8
5	0.149	5340.	0.34	-5000.	0.069	0.259	0.100	22.2
6	0.149	5340.	0.34	-5000.	0.134	0.503	0.100	26.7
7	0.149	5340.	0.34	-5000.	0.176	0.658	0.100	31.1
8	0.149	5340.	0.34	-5000.	0.217	0.812	0.100	35.6
9	0.149	5340.	0.34	-5000.	0.258	0.967	0.100	40.0
TOE					0.080	1.723	0.100	

COEFFICIENT OF RESTITUTION OF SOIL 1.000

OPTIONS AND SPECIFICATIONS

PHI	1.40	S-DAMPING	VISCOUS	RWT (KIPS)	0.00
IOUT	10	P-DAMPING	1	SOIL DIST. NO.	0
IFUEL	1	J SKIN	0.20	TDEL (SEC.)	0.0000
IOSTR	0	J TOE	0.10	TEMAX (MS)	0.00
		TIME INCR. (MS)0.091			

FIGURE 3C-1

RULT= 60.0, AT TOE= 4.8 TONS

TRANSFERRED ENERGY, MAX= 27.7 K-FT
FIN= 27.2 K-FT
TRANSFERRED ENERGY, MAX= 21.1 K-FT
FIN= 20.5 K-FT
TRANSFERRED ENERGY, MAX= 22.9 K-FT
FIN= 22.4 K-FT

TABLE OF EXTREME VALUES FOR PILE AND TIME OF OCCURRENCE

ELEM. NO.	FMAX KIPS	FMIN KIPS	MINSTR KSI	MAXSTR KSI	VELMX FT/S	DISMX INCH
1	269.2(71)	0.0(0)	0.0(0)	27.4(71)	11.99(38)	1.824(289)
2	263.8(72)	0.0(0)	0.0(0)	26.9(72)	12.95(84)	1.804(290)
3	262.7(43)	0.0(0)	0.0(0)	26.8(43)	14.54(82)	1.783(290)
4	263.6(46)	0.0(0)	0.0(0)	26.8(46)	15.24(81)	1.763(293)
5	264.8(49)	0.0(0)	0.0(0)	27.0(49)	14.39(80)	1.744(300)
6	260.4(52)	0.0(0)	0.0(0)	26.5(52)	12.28(79)	1.727(301)
7	242.2(55)	0.0(0)	0.0(0)	24.7(55)	11.92(94)	1.712(301)
8	203.7(57)	0.0(0)	0.0(0)	20.7(57)	13.45(97)	1.701(300)
9	132.4(58)	0.0(0)	0.0(0)	13.5(58)	15.86(64)	1.694(300)

THE MAXIMUM TRANSFERRED ENERGY (ENTHRU) WAS 22.9 K-FT

STROKES ANALYZED AND LAST RETURN (FT) 5.0 3.7 4.0 3.9

FIGURE 3C-2

RULT= 90.0, AT TOE= 7.2 TONS

TRANSFERRED ENERGY, MAX= 22.3 K-FT
FIN= 20.6 K-FT

TABLE OF EXTREME VALUES FOR PILE AND TIME OF OCCURRENCE

ELEM. NO.	FMAX KIPS	FMIN KIPS	MINSTR KSI	MAXSTR KSI	VELMX FT/S	DISMX INCH
1	306.8(69)	0.0(0)	0.0(0)	31.2(69)	14.27(35)	1.354(214
2	304.3(70)	0.0(0)	0.0(0)	31.0(70)	14.13(38)	1.327(215
3	299.6(39)	0.0(0)	0.0(0)	30.5(39)	14.32(79)	1.298(215
4	300.3(43)	0.0(0)	0.0(0)	30.6(43)	15.05(78)	1.269(217
5	303.0(46)	0.0(0)	0.0(0)	30.9(46)	13.86(78)	1.241(219
6	300.0(49)	0.0(0)	0.0(0)	30.5(49)	12.38(50)	1.215(219
7	279.6(52)	0.0(0)	0.0(0)	28.5(52)	12.12(54)	1.192(217
8	232.9(54)	0.0(0)	0.0(0)	23.7(54)	13.74(58)	1.175(218
9	149.8(56)	0.0(0)	0.0(0)	15.3(56)	16.28(61)	1.164(220

THE MAXIMUM TRANSFERRED ENERGY (ENTHRU) WAS 22.3 K-FT

STROKES ANALYZED AND LAST RETURN (FT) 4.8 4.6

FIGURE 3C-3

RULT= 120.0, AT TOE= 9.6 TONS

TRANSFERRED ENERGY, MAX= 20.9 K-FT
FIN= 18.1 K-FT

TABLE OF EXTREME VALUES FOR PILE AND TIME OF OCCURRENCE

ELEM. NO.	FMAX KIPS	FMIN KIPS	MINSTR KSI	MAXSTR KSI	VELMX FT/S	DISMX INCH
1	333.1(67)	0.0(0)	0.0(0)	33.9(67)	15.38(32)	1.075(164)
2	334.5(68)	0.0(0)	0.0(0)	34.1(68)	15.22(35)	1.037(165)
3	320.6(37)	0.0(0)	0.0(0)	32.6(37)	14.99(38)	0.997(166)
4	321.4(40)	0.0(0)	0.0(0)	32.7(40)	14.56(41)	0.956(165)
5	324.1(43)	0.0(0)	0.0(0)	33.0(43)	13.84(44)	0.915(163)
6	320.8(47)	0.0(0)	0.0(0)	32.7(47)	12.86(47)	0.877(159)
7	297.3(50)	0.0(0)	0.0(0)	30.3(50)	12.18(51)	0.846(156)
8	245.1(52)	0.0(0)	0.0(0)	25.0(52)	13.09(55)	0.823(152)
9	156.3(54)	0.0(0)	0.0(0)	15.9(54)	14.97(58)	0.810(150)

THE MAXIMUM TRANSFERRED ENERGY (ENTHRU) WAS 20.9 K-FT

STROKES ANALYZED AND LAST RETURN (FT) 5.2 5.2

FIGURE 3C-4

RULT= 180.0, AT TOE= 14.4 TONS

TRANSFERRED ENERGY, MAX= 17.8 K-FT
FIN= 13.0 K-FT
TRANSFERRED ENERGY, MAX= 21.7 K-FT
FIN= 16.9 K-FT

TABLE OF EXTREME VALUES FOR PILE AND TIME OF OCCURRENCE

ELEM. NO.	FMAX KIPS	FMIN KIPS	MINSTR KSI	MAXSTR KSI	VELMX FT/S	DISMX INCH
1	419.3(98)	0.0(0)	0.0(0)	42.7(98)	18.20(30)	0.890(121
2	408.4(101)	0.0(0)	0.0(0)	41.6(101)	18.03(33)	0.842(122
3	391.0(69)	0.0(0)	0.0(0)	39.8(69)	17.71(36)	0.787(122
4	390.5(86)	0.0(0)	0.0(0)	39.8(86)	17.09(39)	0.730(124
5	412.7(83)	0.0(0)	0.0(0)	42.0(83)	15.99(42)	0.671(124
6	413.9(81)	0.0(0)	0.0(0)	42.1(81)	14.36(45)	0.616(128
7	365.0(83)	0.0(0)	0.0(0)	37.2(83)	12.75(49)	0.576(135
8	300.2(84)	0.0(0)	0.0(0)	30.6(84)	12.51(53)	0.548(136
9	181.2(85)	0.0(0)	0.0(0)	18.4(85)	13.12(56)	0.531(135

THE MAXIMUM TRANSFERRED ENERGY (ENTHRU) WAS 21.7 K-FT

STROKES ANALYZED AND LAST RETURN (FT) 5.4 6.3 6.3

FIGURE 3C-5

RULT= 240.0, AT TOE= 19.2 TONS

TRANSFERRED ENERGY, MAX= 21.0 K-FT
FIN= 13.0 K-FT

TRANSFERRED ENERGY, MAX= 23.2 K-FT
FIN= 15.3 K-FT

TABLE OF EXTREME VALUES FOR PILE AND TIME OF OCCURRENCE

ELEM. NO.	FMAX KIPS	FMIN KIPS	MINSTR KSI	MAXSTR KSI	VELMX FT/S	DISMX INCH
1	515.7(96)	0.0(0)	0.0(0)	52.5(96)	20.96(29)	0.839(104
2	499.3(93)	0.0(0)	0.0(0)	50.8(93)	20.76(32)	0.761(108
3	486.8(89)	0.0(0)	0.0(0)	49.6(89)	20.36(35)	0.687(111
4	491.8(85)	0.0(0)	0.0(0)	50.1(85)	19.56(38)	0.616(114
5	510.4(81)	0.0(0)	0.0(0)	52.0(81)	18.08(41)	0.539(115
6	501.0(79)	0.0(0)	0.0(0)	51.0(79)	15.82(44)	0.464(117
7	416.7(79)	0.0(0)	0.0(0)	42.4(79)	13.37(48)	0.398(120
8	320.0(83)	0.0(0)	0.0(0)	32.6(83)	12.01(51)	0.349(124
9	195.3(53)	0.0(0)	0.0(0)	19.9(53)	11.34(54)	0.323(126

THE MAXIMUM TRANSFERRED ENERGY (ENTHRU) WAS 23.2 K-FT

STROKES ANALYZED AND LAST RETURN (FT) 6.9 7.4 7.4

FIGURE 3C-6

FRICTION PILE

SUMMARY

NO	RULT TONS	BLOW CT BPF	STROKE FT	MIN STR KSI	MAX STR KSI	BLOWS/ MINUTE	B.C. PR. PSI	FUEL REQ.
1	60.0	8.	3.99	0.00	27.42	58.1	N/A	N/A
2	90.0	11.	4.79	0.00	31.24	53.3	N/A	N/A
3	120.0	17.	5.19	0.00	34.06	51.4	N/A	N/A
4	180.0	28.	6.34	0.00	42.69	46.7	N/A	N/A
5	240.0	54.	7.35	0.00	52.52	43.5	N/A	N/A

FIGURE 3C-7

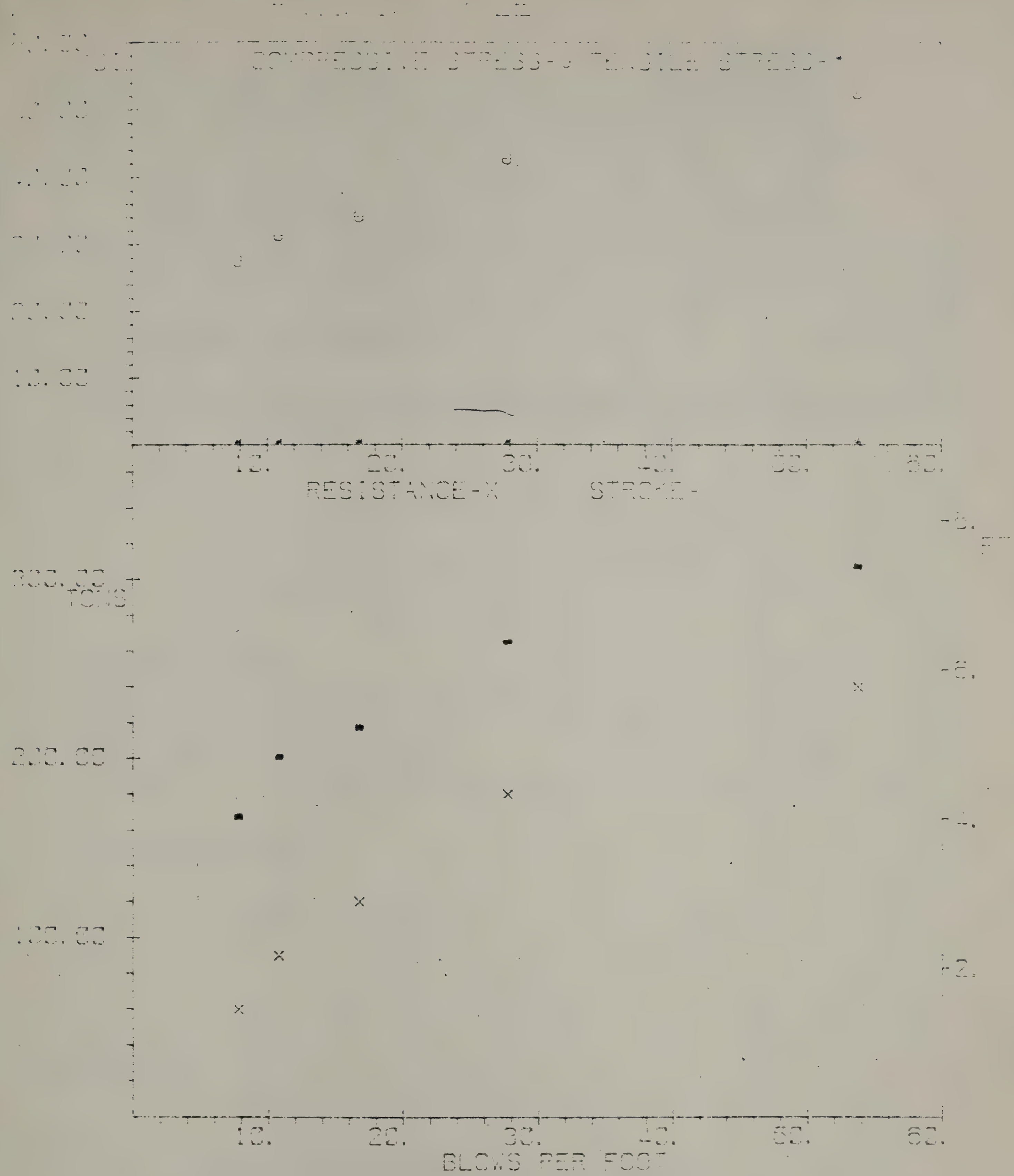
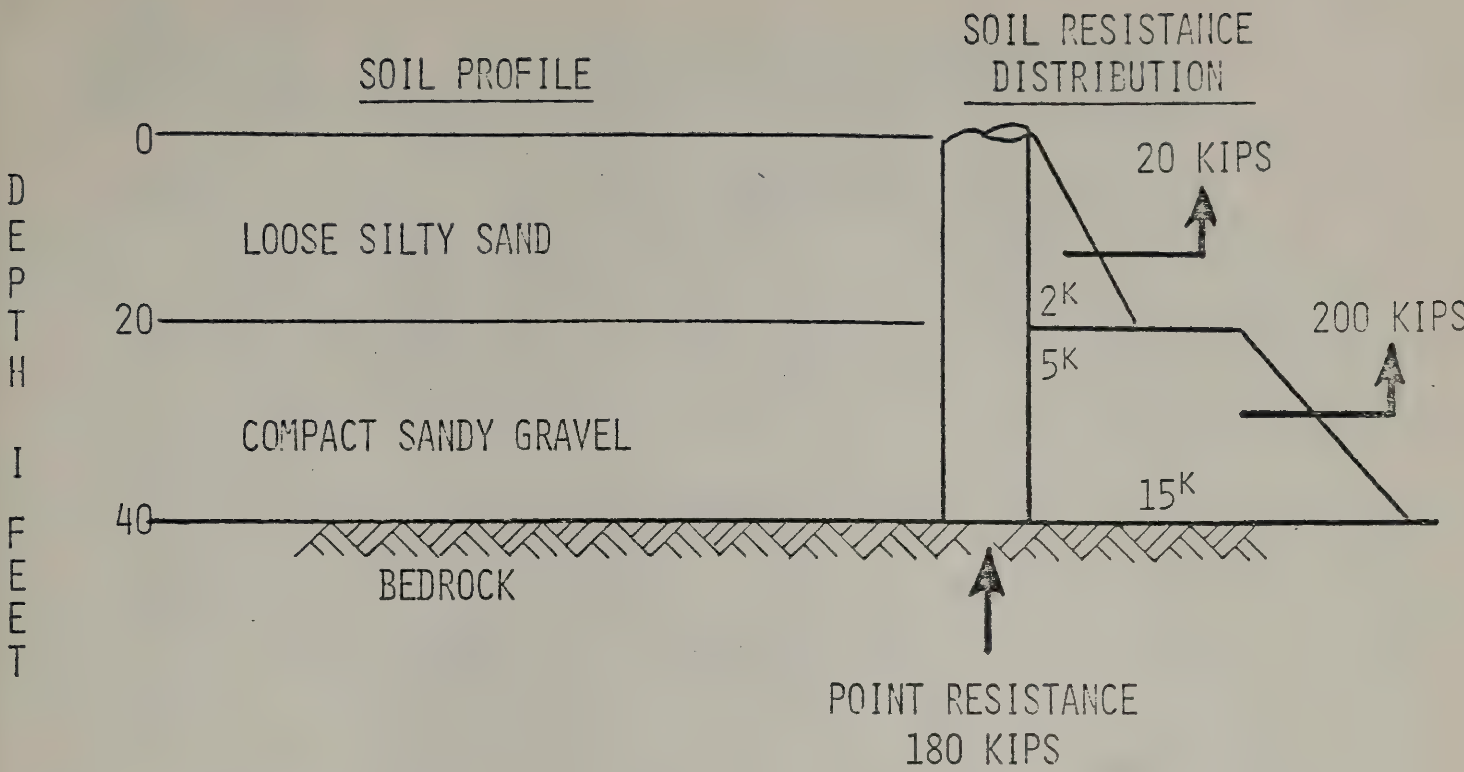

RESISTANCE-X
STROKE-
TONS
10.
20.
30.
40.
50.
60.
100.00
200.00
BLOWS PER FOOT

FIGURE 3C-8



PILE DESIGN LOAD		100 TONS
ULTIMATE SOIL RESISTANCE (SAFETY FACTOR = 2)		200 TONS
LENGTH FOR 200 TON RESISTANCE		40 FEET
PILE TYPE - - - 16 INCH DIAMETER PIPE,		0.344 INCH WALL

CASE DAMPING VALUES			
	SKIN	LAYER 1 LAYER 2	USE 0.2
	TOE		0.1
QUAKE VALUES	SKIN		0.1
	TOE		0.1

DESIGN DATA FOR WAVE EQUATION ANALYSIS

END BEARING PILE EXAMPLE

FIGURE 4A

INPUT CARD REFERENCE NO.	DESIGNATION OF NECESSARY INPUT				
			TITLE		
1.0			END BEARING PILE		
	IOUT	IHAMR	IPERCS		
2.0	10, ,	4, ,,,,,,,	-92 *		
	WT. CAP		STIFF-CB		
3.0	1.2,		21,000., *		
		C.O.R.-CB			
4.0	,	0.8, *			
	LENGTH	AREA-PILE TOP			
5.0	40.0,	16.9, *			
	QUAKE-SKIN	QUAKE-TOE	DAMP-SKIN	DAMP-TOE	RULT
6.0	0.1,	0.1,	0.2,	0.1,	-1.0,*
	X_{PI}	% SOIL RESISTANCE			
6.401	0.0	0.0			
6.402	20.0	2.0			
6.403	20.0	5.0			
6.404	40.0	15.0			
7.0	BLANK				
8.0	BLANK				
	ULTIMATE RESISTANCE				
9.0	120.0,	160.0,	200.0	250.0	. 300.0 *

CASE WAVE EQUATION INPUT

END BEARING PILE EXAMPLE

FIGURE 4B

END BEARING PILE

PILE DESCRIPTION

X BEL. TOP (FT)	0.0	40.0
A (SQ. IN.)	16.9	16.9
E (KSI)	29000.	29000.
GAMMA (LB/CU FT)	492.0	492.0

HAMMER MODEL DEL. D-22

ELEMENT NUMBER	WEIGHT (KIPS)	STIFFNESS (K/IN)	COEFF. RESTITUTION
1	1.213		
2	1.213	191666.7	
3	1.213	191666.7	
4	1.213	191666.7	
ANVIL	1.595	101833.3	0.850
CAP	1.200	21000.0	0.800
CUSHION		0.0	1.000
PILE TOP			0.850

PILE PROPERTIES

PILE LENGTH= 40. FT., AREA(AT TOP)= 16.9 S-IN
E. MODUL(AT TOP)=29000. KSI., SPEC. WT.(AT TOP)= 492. LBS/CU FT

NO.	WEIGHT (KIPS)	STIFFN. (K/IN)	PDAMP. (KS/FT)	SPLICE (KIPS)	SOIL-S (PCT.)	SOIL-D (KS/FT)	QUAKE (IN.)	L.B.T. (FT.)
1	0.257	9189.	0.59	0.	0.011	0.073	0.100	4.4
2	0.257	9189.	0.59	-5000.	0.034	0.220	0.100	8.9
3	0.257	9189.	0.59	-5000.	0.057	0.366	0.100	13.3
4	0.257	9189.	0.59	-5000.	0.080	0.513	0.100	17.8
5	0.257	9189.	0.59	-5000.	0.102	0.659	0.100	22.2
6	0.257	9189.	0.59	-5000.	0.125	0.806	0.100	26.7
7	0.257	9189.	0.59	-5000.	0.148	0.952	0.100	31.1
8	0.257	9189.	0.59	-5000.	0.170	1.098	0.100	35.6
9	0.257	9189.	0.59	-5000.	0.193	1.245	0.100	40.0
TOE					0.080	2.966	0.100	

COEFFICIENT OF RESTITUTION OF SOIL 1.000
SKIN FRICTION CONSTANT FOR ALL RULT VALUES

OPTIONS AND SPECIFICATIONS

PHI 1.40	S-DAMPING	VISCOUS	RWT (KIPS)	0.00
IOUT 10	P-DAMPING	1	SOIL DIST. NO.	0
IFUEL 1	J SKIN	0.20	TOEL (SEC.)	0.0000
IOSTR 0	J TOE	0.10	TEMAX (MS)	0.00
	TIME INCR. (MS)	0.091		

FIGURE 4C-1

RULT= 120.0, AT TOE= 9.6 TONS

TRANSFERRED ENERGY, MAX= 17.5 K-FT
FIN= 15.9 K-FT

TRANSFERRED ENERGY, MAX= 20.6 K-FT
FIN= 19.1 K-FT

TABLE OF EXTREME VALUES FOR PILE AND TIME OF OCCURRENCE

ELEM. NO.	FMAX KIPS	FMIN KIPS	MINSTR KSI	MAXSTR KSI	VELMX FT/S	DISMX INCH
1	507.5(30)	0.0(0)	0.0(0)	30.0(30)	15.09(31)	0.897(166)
2	512.0(33)	0.0(0)	0.0(0)	30.3(33)	14.87(34)	0.876(171)
3	511.0(36)	0.0(0)	0.0(0)	30.2(36)	14.53(37)	0.857(178)
4	504.0(39)	0.0(0)	0.0(0)	29.8(39)	14.08(40)	0.840(178)
5	491.4(42)	0.0(0)	0.0(0)	29.1(42)	13.51(43)	0.824(178)
6	473.3(45)	0.0(0)	0.0(0)	28.0(45)	12.87(46)	0.808(180)
7	443.6(48)	-4.4(66)	-0.3(66)	26.2(48)	12.79(50)	0.795(190)
8	375.2(51)	-35.3(65)	-2.1(65)	22.2(51)	15.08(55)	0.791(189)
9	245.8(52)	-28.9(64)	-1.7(64)	14.5(52)	18.34(57)	0.788(189)

THE MAXIMUM TRANSFERRED ENERGY (ENTHRU) WAS 20.6 K-FT

STROKES ANALYZED AND LAST RETURN (FT) 5.0 5.7 5.5

FIGURE 4C-2

RULT= 160.0, AT TOE= 49.6 TONS

TRANSFERRED ENERGY, MAX= 22.0 K-FT
FIN= 18.9 K-FT
TRANSFERRED ENERGY, MAX= 21.1 K-FT
FIN= 18.0 K-FT

TABLE OF EXTREME VALUES FOR PILE AND TIME OF OCCURRENCE

ELEM. NO.	FMAX KIPS	FMIN KIPS	MINSTR KSI	MAXSTR KSI	VELMX FT/S	DISMX INCH
1	557.1(29)	0.0(0)	0.0(0)	33.0(29)	16.90(30)	0.772(132)
2	561.9(32)	0.0(0)	0.0(0)	33.2(32)	16.64(33)	0.741(133)
3	560.6(35)	0.0(0)	0.0(0)	33.2(35)	16.24(36)	0.709(130)
4	553.1(38)	0.0(0)	0.0(0)	32.7(38)	15.73(40)	0.678(129)
5	540.7(42)	0.0(0)	0.0(0)	32.0(42)	15.11(43)	0.648(146)
6	524.4(45)	0.0(0)	0.0(0)	31.0(45)	14.41(46)	0.625(145)
7	496.5(48)	0.0(0)	0.0(0)	29.4(48)	14.07(49)	0.602(142)
8	435.3(50)	0.0(0)	0.0(0)	25.8(50)	15.45(53)	0.583(141)
9	318.5(52)	0.0(0)	0.0(0)	18.8(52)	18.48(57)	0.567(146)

THE MAXIMUM TRANSFERRED ENERGY (ENTHRU) WAS 21.1 K-FT

STROKES ANALYZED AND LAST RETURN (FT) 6.8 6.2 6.2

FIGURE 4C-3

RULT= 200.0, AT TOE= 89.6 TONS

TRANSFERRED ENERGY, MAX= 19.2 K-FT
FIN= 15.3 K-FT

TABLE OF EXTREME VALUES FOR PILE AND TIME OF OCCURRENCE

ELEM. NO.	FMAX KIPS	FMIN KIPS	MINSTR KSI	MAXSTR KSI	VELMX FT/S	DISMX INCH
1	555.8(29)	0.0(0)	0.0(0)	32.9(29)	16.72(30)	0.655(119
2	560.8(32)	0.0(0)	0.0(0)	33.2(32)	16.45(33)	0.621(118
3	559.7(35)	0.0(0)	0.0(0)	33.1(35)	16.05(36)	0.587(115
4	552.5(38)	0.0(0)	0.0(0)	32.7(38)	15.53(40)	0.552(116
5	540.7(42)	0.0(0)	0.0(0)	32.0(42)	14.91(43)	0.514(118
6	524.6(45)	0.0(0)	0.0(0)	31.0(45)	14.19(46)	0.484(106
7	498.1(48)	0.0(0)	0.0(0)	29.5(48)	13.72(49)	0.458(105
8	444.2(50)	0.0(0)	0.0(0)	26.3(50)	14.26(53)	0.433(104
9	358.4(53)	0.0(0)	0.0(0)	21.2(53)	15.93(57)	0.410(103

THE MAXIMUM TRANSFERRED ENERGY (ENTHRU) WAS 19.2 K-FT

STROKES ANALYZED AND LAST RETURN (FT) 6.4 6.6

FIGURE 4C-4

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